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98 Design of a Water Tank. Yarnall '01.

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20 May '48. Sen. g. Univ. Y. David Robert

PREFACE.

The following design of a hemispherical bottom water tank of 40,000 gallons capacity was made by the writer in the regular course of business while in the employ of Coatesville Boiler Works in the capacity of Mechanical Engineer.

There is no particular claim to originality in this design, in as much as structures of similar shape have been designed before, although the acceptance of this type of tank by the trade generally is comparatively recent. The writer has introduced certain minor details into the design however which he believes tend toward improvement over similar structures that have come to his notice. These questions will be apparent upon reading the pages that follow, and upon examining the accompanying

drawings which later were made by the writer.

The writer has made free use of " Tanks & Towers for Water Works " by J. M. Hazelhurst a book published in 1904, which is perhaps the most comprehensive work on this subject. Use has also been made of Johnson's " Modern Frame Structures ", Merrimans " Mechanics of Materials ", and Kent's, Trautwine's, Carnegie's, Lukens' and Pencoyd's hand-books. The writer has also studied, whenever possible, structures of similar design that have been built and are at the present time in actual service. Information relative to quality of steel, shop methods and best practice in riveting and caulking has been obtained from Coatesville Boiler Works. This data, the result of years of experience in heavy plate metal construction, is of great value to the designer.

The design of another hemispherical bottom tank is also submitted here, but its details

have not been worked up for this thesis because it is possible that this tank may not be built. The drawing was a hasty design gotten up for estimating purposes only. The tank has a capacity of 408,000 gallons, and is as far as known larger than any hemispherical bottom tank that has ever been built.

Respectfully submitted,

D. Robert Yarnall.

May 1st 1905

General.

The two fundamental principles of engineering design are stability and economy.

First the structure must safely withstand all the stresses which are applied to it.

Second the structure must be built and maintained at the lowest possible cost.

It is because of the above two points of superiority that the hemispherical bottom water tower has come into such prominent use in the last few years, supplanting in many cases the flat bottom storage tanks made of wood and steel and the steel cylindrical stand-pipe.

The design, which we are about to consider, is of a cylindrical steel tank, as per the accompanying drawing, having a self supporting steel bottom made hemispherical in shape and riveted to the lower end of the cylindrical portion of the tank. The tank is supported on

four steel columns securely riveted to the bottom ring of cylindrical portion all as shown on the accompnying drawing. The tank is provided with a steel roof supported on an angle iron frame work.

At a glance the merits of this form of storage tank and tower are apparent.

In the first place it is cheaper to build than a flat bottom tank of the same capacity, because one can dispense with the heavy expensive platform supporting the bottom of the flat bottom tanks made of wood or steel. This platform of necessity must be made heavy because of the load per square foot which it must support and the flat bottom of the tank itself in no way supports its superimposed load.

In the second place all parts of the tank are exposed at all times, hence, if any leaks occur for any cause they can be readily detected and caulked tight, without in any way dismantling

the structure or even emptying the tank. Too much stress cannot be laid upon this great advantage of the hemispherical bottom over the flat bottom tank.

No matter how carefully a water tank may be constructed there comes a time some day when a leak developes, because of unequal expansion or from some other cause. If this leak is inaccessible, it will in all probability be neglected until serious damage is done to the tank. In the case of the hemispherical bottom tank, however, all parts are accessible at all times for careful inspection; and they can be readily caulked, cleaned and painted as required. With proper care and attention the life of these tanks is many times the life of all other forms of tank.

It might be noted here that the reinforced concrete tank has not been considered in this discussion, in view of the fact that its use commercially has been very limited up to the

present time.

The respective merits of steel and wood in water storage tanks will not be considered here, suffice it to say that in the writer's opinion the steel tank with intelligent care will outlive a wooden tank of the same capacity. The cost of the two styles of storage tanks is about even, if anything, a little in favor of the steel tank.

Comparing the hemispherical bottom steel storage tank with a cylindrical steel stand-pipe it will be found that the former design has many points of advantage.

The advantages of storing water at an elevation are uniformly recognized, the cost of continuous pumping being thereby reduced. Direct pumping systems, in addition to extra cost of operating, are unsatisfactory due to fluctuating pressure, causing serious disturbance in mains and service tanks. To obtain satisfactory pressure

for most city and industrial purposes water must be stored above an elevation of 75 feet (33 lbs. pressure per square inch) and in most cases above 100 feet (43 lbs. pressure per square inch). The advantage of the elevated tank over the stand-pipe for this purpose, is that, for the same cost of structure, from two tb three times as much water can be stored above this elevation.

" Considering for a moment the municipal water problem, a stand-pipe should be constructed only where it can be located on a natural elevation. Due investigation should be given to the cost of reaching such a location, bearing in mind that the largest main must connect the pump with the stand-pipe. It has often been found advantageous in practice to build a water tower of increased height, and place the same adjacent to the pumping station even when natural elevations exist elsewhere."

Considering the industrial water problem where the demand is for a comparatively small storage capacity there is no form of tank which so efficiently and economically adapts itself to the varied requirements of the factory water supply as the hemispherical bottom tank and tower. At a moderate cost one can store a large quantity of water at a height sufficient to maintain for a considerable time a practically constant head on fire mains, sprinkler systems, or the like.

As the outlet for these tanks is always at the extreme bottom of the hemisphere, any sediment which would tend to collect on the bottom in continually carried away when the outlet is opened, thus at all times keeping the tank clean on the inside and consequently reducing still further the cost of maintenance.

Another advantage of the hemispherical bottom water tank and tower is in the fact that it is a structure decidedly more artistic than the

plain cylindrical stand-pipe, or the flat bottom tank supported on a tower, because its lines are graceful in themselves giving with all an effect of stability to the structure which is most pleasing.

PROBLEM.

The hemispherical bottom storage tank and tower in question is to have a capacity of 40,000 gallons. The top of the tank proper is to be 70 feet above the ground level. The tank is to be provided with a steel conical roof and is to be erected complete on foundations designed by the builder of tank but built by the purchaser at Jersey City, N. J.

The above meagre data was given the writer from which to design the tank.

GENERAL DATA.

A tank elevated on a steel tower, in the writer's opinion, looks much better if its vertical height is considerably greater than its diameter. It was decided in this case, to give the best effect, to use a diameter of 16 feet; then in order to get a capacity of 40,000

gallons it was found necessary to make the cylindrical portion of the tank 22 feet high . In other words we find, -

Volume of cylinder $= 22 \times \pi \times (8)^2$ = 4422 cu. ft.

Volume of hemispherical bottom $= \frac{\pi D^3}{12}$ = 1072 cu. ft.

Total volume in cu. ft. = 5494

Total capacity in U. S. gallons = 41095

It will be noticed that this capacity is figured about 1,000 gallons over the capacity required, namely, 40,000 gallons. In a storage tank it is always best to allow six or eight inches additional height because of the fact that the tank will never be filled up to the brim on account of the overflow. In this case I have allowed eight inches additional height which gives an additional 1,000 gallons capacity to the tank at very little additional cost.

TOWER DATA.

We will support this tank on four 10" latticed channel columns,battered away from the

vertical centre line through the tank, to give a broader base thus increasing the stability of the structure.

The latticed channel column has been decided on in this case because of the fact that it is built of shapes at all times rolled by the mills, hence, no delay will be caused in getting the material. But principally it is chosen because it is an open column and all parts are at all times accessible for thorough inspection and painting. Then it is a column that is readily attached to the tank, and a column to which struts and lateral bracing can be securely and cheaply attached, all as hereinafter described.

These columns will be tied together, as will be seen from the accompanying drawing, every 16 feet of vertical height by means of struts each composed of four 2" x 3" x 1/4" angles latticed together as per drawing. This form of strut has been chosen because it is an open strut,

at all times accessible for painting, and because it is an economical member to build.

The tower will be further braced by the use of two 1" round diagonal braces in each square formed by the horizontal struts and columns, making twenty-four rods in all. Each end of each rod will have attached a standard steel screwed clevis as manufactured by Edge Moor Iron Works.

It has been found in practice that it is cheaper and decidedly more reliable to drop forge the clevis and then to screw it on to the end of the rod, rather than to form the clevis and then weld it to the rod.

Each rod is also to be provided near one end with a standard turn-buckle to take up any derangement due to possible settlement of any one of the four column foundations.

It is to be noted that the ends of these diagonal tie rods have not been upset so as to make the diameter at the root of the thread equal

the diameter of the rod, because it is cheaper in rods up to a diameter of about 1-1/2" to use a rod of larger diameter the whole length, the additional weight of material being less expensive than the work of upsetting the ends.

TANK DESIGN.

The pressure exerted by one cu. ft. of water (or 62-1/2 lbs.) over an area of 144 sq. in. = .434 lbs. per sq. in., hence to find the pressure in a tank at any point multiply the head at this point (in feet) by .434 and it will give the pressure per sq. in. tending to burst the tank.

The tank in question is 16 feet in diameter and 22 feet high in the straight, as previously mentioned. In order to determine the thickness of the plates composing this straight part of the tank we can use the following

formula:- $(pd = 2tS)$ See Merriman also.

$$\text{Thickness} = \frac{\begin{matrix}\text{pressure} \\ \text{per sq. in.}\end{matrix} \times \begin{matrix}\text{radius} \\ \text{in inches}\end{matrix} \times \begin{matrix}\text{factor} \\ \text{of safety}\end{matrix}}{\begin{matrix}\text{tensile} \\ \text{strength}\end{matrix} \times \begin{matrix}\text{efficiency} \\ \text{of the joint.}\end{matrix}}$$

The derivation of this formula is very simple and may be found in any text book on mechanics of materials. We have quoted direct from Trautwine's handbook where this derivation is concisely and briefly made.

Take into consideration the joint at the bottom having greatest pressure on it, which would mean a joint 22 feet down from the top of tank and substituting in our formula we get:-

$$\text{Thickness} = \frac{(.434 \times 22') \times 96'' \times 5}{60000 \times .55} = .14''$$

Hence, we find that the thickness of plate which is thick enough to withstand this internal pressure of about 9-1/2 lbs. per sq. in. is much too thin to use in practice first, because

the tank has to withstand wind pressure when empty; second, because of the roof load which it has to support; third, because reasonable allowance should be made for corrosi~~ve~~on; and fourth, because it is practically impossible to keep such thin plates tight at riveted joints ,because of the difficulty of caulking. This latter point might however be overcome by using smaller rivets spaced very close together, but this is impracticable in our case.

It has been found impossible to make good tight caulked single riveted joints with plates much less than 1/4" thick. We will therefore use in the straight 22 feet of this tank 1/4" plates excepting in the bottom ring which will be made 3/8" thick for reasons to be hereinafter explained. Standard steel rivets 3/4" diameter will be used pitched 2-1/2" Centre to Centre. It is the best practice in work that has to be caulked to pitch all rivets at a distance equal

to ten times the thickness of the thinnest plate. In our case this would be, 10 x 1/4" = 2-1/2", which justifies our use of the above pitch, namely, 2-1/2".

The single riveted seam having an efficiency of 55% of the strength of the plate itself, has been used in our formula, because it is the cheapest form of seam to make in the shop. If however, it had been found that by using this efficiency we would have to use a thickness of plate in excess of what would be termed good practice, we would have used a double staggered riveted seam having an efficiency of from 71% to 75% if properly proportioned. In some cases where it is desired to keep the weight of the structure very low, and still maintain the required strength, a properly proportioned triple riveted butt strap seam having inside and outside welt strips and an efficiency of 85% to 87-1/2% would be used.

We have above, considered a point on a

vertical seam 22 feet down from the top of the tank where we have the greatest pressure. We have found that it will not be good practice to use smaller rivets than 3/4" or to use a greater pitch than 2-1/2", therefore, on all the vertical seams in the remainder of the tank we will use the same size of rivets and the same pitch.

We will next consider the girth seams. These seams are subjected, there being no appreciable stress on them due to the internal water pressure, to the weight above them which consists in the weight of the plates forming the cylinder, the roof and its supporting structure, and the roof load composed of the snow and wind loads. For the design of these girth seams we will assume that there is exerted on each and every square foot of horizontal projected area of the roof, a vertical load of 50 lbs. per sq. ft. due to the wind and snow, hence we find:-

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Snow and wind load = 15700 lbs.

Weight roof = 4970 lbs.

Weight 3 rings of shell, all 1/4" thick = 10335 lbs.

It will be noted that we are considering the third girth seam down from the top of tank rather than the fourth seam, because the fourth seam has other stresses which must be taken into consideration, and the design of it will be considered later.

We therefore find the total dead load on the girth seam in question tending to shear the rivets = 31000 lbs.

It will be noted that this dead load is distributed practically uniformly around the circumference, hence, the load per lineal inch

$$= \frac{31000}{2 \times \pi \times 8 \times 12} = 51 \text{ lbs. per lineal inch.}$$

In addition to this we have a load due to the wind acting on the side of the tank tending to shear the rivets of the seam in question. This load is best determined by considering our tank a cantilever beam the point of support being at the seam in question. Then the bending moment M due to the wind, is the pressure per square foot on one-half the vertical projection of that part of the tank above the girth seam in question, times the distance above the seam to the centre of pressure of the wind or :-

$$M = 50 \times \frac{dh}{2} \times \frac{h}{2} = 12.5\ dh^2$$

This moment causes a maximum stress in shell of the tank (the extreme fibre) equal to:-

$S' = \frac{MC}{I}$

C = radius = 8'

$I = 1/4\pi(r_1^4 - r_2^4) =$ approx. $\frac{t}{12} x \pi r^3$

r, r_1, r_2 , are measured in feet.

I = moment of inertia.

r_1 = outer radius in feet.

r_2 = inner radius in feet.

r = radius of tank in feet.

t = thickness in inches.

h = distance in feet

down from top of tank to seam under consideration.

Hence in lbs. per sq. in.

$$S' = \frac{1}{144} \times \frac{Mr}{\frac{\pi t r^3}{12}} = 1.33 \frac{h^2}{td}$$

And the stress per lineal inch along the circumferential seam due to this bending moment occasioned by the wind

$$S_, = 1.33 \frac{h^2}{t\ d} \times t = 1.33 \frac{h^2}{d} = 1.33 \frac{\times 18 \times 18}{16} = 27 \text{ lbs.}$$

Adding this load to the load found above, namely, 51 lbs. per lineal inch due to the weight of tank itself including the roof, snow and wind loads, we get for the total stress per lineal inch of the circumferential seam under consideration 78 lbs.

The shearing value of one 3/4" rivet (taking allowable shear = 7500) = 3310 lbs.

Hence, we find that the necessary pitch for rivets in this seam to support the loads above enumerated will be :-

$$\frac{3310}{78} = 42.4''$$

This pitch is much in excess of what will be used, because our tank is to hold water and in order to do so without leaking the rivets must be spaced close together so that the plates can be properly caulked. As previously stated it has been found in practice that in riveting 1/4" thick plates 3/4" rivets should be used and pitched about 2-1/2" Centre to Centre.

Having found that this third seam is properly riveted and as the same thickness of plates has been used throughout the cylinder, we will use the same single riveted seam for the remaining girth seams.

HEMISPHERICAL BOTTOM.

We will now design the hemispherical bottom.

Upon consideration it has been found that the thickness of the plates in a hemispherical bottom need be but little more than one-half that of the bottom ring [illegible] of the cylindrical portion of the tank to which the hemisphere is to be riveted. The computed stress on the bottom plate is only one-half that in the sides of the cylinder subjected to the same internal stress, or it is $\frac{pr}{2}$ per lineal inch where p is the internal pressure per square inch and r is the radius of the bottom in inches. We have found that in the cylindrical portion of our tank, 1/4" plates have a thickness in excess of that required to withstand the pressure due to the water inside the tank and to the external load placed upon them. We have also found that

single riveted joints properly proportioned are amply strong to withstand the loads imposed. In view of the fact that the stresses in the hemispherical bottom are due alone to the internal pressure, we are perfectly justified in using the same single riveted joints and the same thickness of plates; hence, we shall use for the bottom, steel plates 1/4" thick; and we shall use 3/4" rivets spaced about 2-1/2" Centre to Centre.

In regard to the joint between the hemisphere and the cylinder we have to contend alone with the weight of the water in the tank and the weight of the hemispherical bottom. This is a vertical load tending to shear the rivets around the circumferential seam joining the bottom with the sides of tank, then :-

Let W = equal this load

$$w = \frac{W}{2\pi R}.$$

$$w = \frac{\text{weight of water} + \text{weight of bottom}}{2\pi \times \text{radius of tank}}$$

Substituting in this formula we get:-

$W = \frac{387000 + 18000}{2 \pi \text{ x } 8 \text{ x } 12} = 675.$ lbs. per lineal inch of circumference.

By referring to the drawing it will be noticed that this seam is a double riveted seam having 3/4" rivets pitched about 3" on centres; its efficiency is 70%.

The thickness of the plates as above is 1/4". The resisting power of our joint per lineal inch together with superimposed load is then $= \frac{1/4 \text{ x } 60,000 \text{ x } 70\%}{4} = 2625$, but our load was only 675 lbs. per lineal inch of circumference, and therefore it will appear that it was unnecessary to use a double riveted seam at this joint. There are other stresses, however, that have to be taken into consideration, which will be considered later, which necessitates our double riveting this seam.

The location and number of the seams are shown clearly on the accompanying drawing. The lower part of the bottom will be formed of a 1/4" plate~~s~~ dished to ~~a~~ the radius of the bottom, namely, 8 feet and having an external diameter when dished of about 8 feet; to this plate will be riveted at intervals shown the remaining plates which form the surface of the hemisphere. Each one of these latter plates will be dished to the same radius, namely 8 feet, either by means of a steam hammer, or a hydraulic press having a die made to conform with the curvature of the bottom of the tank. Each one of these plates will be riveted to the one next adjoining it and to the lower circular dished plate by means of a single row of 3/4" rivets spaced 2-1/2" on centres as above noted. It is to be noted that these plates will be dished cold.

Riveted to the centre of the extreme bottom portion of the tank will be a cast iron

expansion joint all as explained and detailed in the accompanying drawing.

Design of Tower.

According to J. N. Hazelhurst, whose deductions are lucid and brief, the determination of the stress produced in the tower by the action of the wind is arrived at as follows:-

" Taking the wind in the direction of a diagonal as shown by the arrow W (figure A) the maximum tension is produced in column A, and the maximum compression in column C. Therefore taking moments about mn, W x H or $M = V_a \times X \sin 45^\circ + V_c \times X \sin 45^\circ$; and as $V_a = V_c$ therefore :-

$$V_a = \frac{M}{2X \sin 45^\circ} = .707 \times \frac{M}{X} .$$

This vertical force V_a is resolved into its components, a force downward throught the columns and the horizontal forces which is in turn resolved into two forces lying in the planes of

the trusses A B and A D. The same is ture of V_c at column C, hence, tension in A is $S = V_a \sec \theta$, θ being angle of inclination of post. Compression in C is the same as tension in A.

" The horizontal force then is $h = V_a \tan \theta$, and its components in the sides of the tower marked R (figure B) are $R = h \sin 45^\circ$.

" Substituting the value of h just found we get $R = V_a \tan \theta. (.707)$.

" Having the shear at the top of each post, which is $\frac{W}{4}$ acting in the same direction as the wind, this must be resolved into forces lying in the planes of the sides of the tower. These are equal to $P = \frac{W}{4} \times \sin 45^\circ = \frac{W}{4} (.707)$ acting in the direction shown by the arrow P (figure C).

These forces must be combined with the force R, and this combination is shown in figure D."

It is not necessary to consider here the case where the wind is exerted on the side of

the tower midway between two posts because it is at once apparent, in considering diagram A, that the case above considered gives the maximum stress in the tower members.

By substituting the values in the various formula above given we find that:-

$$V_a = .707 \times \frac{M}{X}$$

M = overturning moment due to the wind = 30 lbs. per sq. ft. x the projected area x 11' (the vertical distance from the centre of gravity of the projected area to the top of the posts) = 165330 ft. lbs.

Substituting these values in the above formula we get:-

$$V_a = 10300 \text{ lbs.}$$

$$V_a = (\text{tension}) = V_c \text{compression.}$$

θ = inclination of posts toward the centre of the figure.

$$\tan\theta = \frac{6.2}{48} = 1.29 \; .129$$

Therefore $\theta = 7^\circ\ 20'$

$$S = V_a \times \sec\theta = 10384$$

$$h = V_a \tan\theta = 1329$$

$$R = h \sin 45^\circ = 940$$

Horizontal shear $\frac{W}{4} = 3757.5$

$P = \frac{W}{4} \sin 45^\circ = 2657$ lbs.

$P - R = 1717$ (see diagram D)

$$(P - R) + P = 4374.$$

For convenience we have tabulated these values on diagram E.

We are now in position to analyze graphically each side of the tower giving the stress in the struts and bracing, and additional stress in columns. Combining these stresses algebraicly we obtain the maximum.

It is unnecessary here to go into a detail explanation of the graphic alalysis of these stresses, as the method of analysis is shown on the drawing.

COLUMN DESIGN.

Upon referring to the stress diagram , we find that the greatest compressive stress in any one column due to the wind load, occurs in the lower section of the column furthest from the wind and = 39144 lbs.

We will now consider the design of the

column. We have found that the direct stress due to the weight of the water in the tank, the weight of the tank itself, together with the roof and the roof load acting down ~~the~~ Each post~~s~~ is 100,000 lbs., adding this to the 39,134 lbs. which is the greatest wind load, we get a total load acting down the lower section of post = 139,134 lbs. Now the unsupported length of column (referring to the drawing) is 16 feet. We will use a column composed of two 10" latticed channels as previously mentioned. Carnegie's handbook gives for two 10" channels 25 lbs. to the foot, a radius of gyration = 3.52; hence, we find $\frac{l}{r} = \frac{16}{3.52} = 4.54$. We find that the ultimate strength of a column having $\frac{l}{r} = 4.54$ is 46200. As we have determined to use a factor of safety of five in the design of this tower, we get by dividing this ultimate strength by the factor of safety an allowable safe resistance of 9240 lbs. per sq. in. Our total load as above was 139,134 lbs. the area

of one 10" x 25 lb. per foot channel is 7.35 or the total area of our column is 14.7 sq. in. ignoring the lattice bars. Dividing our total load by the total number of square inches we get 9470 lbs. as the actual maximum load per square inch acting on the section of column. It will be noted that this load is a little in excess of the allowable load, namely, 9240 lbs. which was determined by using a factor of safety of five We shall, however, use for our column, throughout its length, two 10" 25 lb. per foot channels latticed together by means of standard lattice bars as dimensioned in Pencoyd's handbook.

We will next consider the design of the top of our column. There are many different ways of attaching our column to the tank, but the writer is of the opinion that the method shown on the accompanying drawing is as secure and as easy to build as any method yet devised. It consists in simply riveting two bent plates

to the sides of the top of each column and then in turn securely riveting these plates to the tank. The number of rivets required was determined as follows:-

We have found that the greatest vertical load at the top of each column tending to shear the rivets is approximately 98000 lbs. We shall use 3/4" rivets for riveting these plates to the tank. We shall also use in figuring the number of rivets a very safe allowable shear per square inch, namely, 6000 lbs. The area of one 3/4" rivet is .4418; multiplying this area by the allowable shear per square inch, we get as the allowable shear per rivet 2650 lbs. ~~per sq. in.~~ Dividing this allowable shear per rivet into the total load of 98000 lbs. we get 37 rivets required to resist the shear above mentioned. We shall, however, use in this connection 42 rivets in all, 21 rivets on each side of each post. The same number of rivets is used to rivet

bent plates to ~~column~~ tank. We have also investigated these rivets for bearing and find they have the proper diameter for thickness of plates used.

HORIZONTAL STRUT DESIGN.

Again referring to the diagram of wind stresses, we find that the greatest load on any horizontal strut comes on the lower strut and is equal to a compression of 5225 lbs. As explained early in this description, we will use for these struts four angle irons latticed together. Now proceeding exactly as we did in the case of the column design, finding a value of $\frac{l}{r}$, then from the table getting the value of the ultimate allowable stress, dividing this by the factor of safety and obtaining thereby a value for the allowable stress per square inch in the strut, Then dividing this value into the total load on the strut we get the number of square inches required in the strut to withstand the load imposed.

We find that it will be necessary to use four 3" x 2" x 1/4" angles latticed together as per drawing. The size and spacing of lattice bars, as in the case of column, to be according to Pencoyd standard.

TIE ROD DESIGN.

Again consulting our stress diagram we find that the greatest stress in any tie rod is = 7125 lbs. tension and occurs, as will be noted, in the upper tie rods. Taking an allowable stress per square inch = $\frac{60,000}{5}$ = 12000 lbs. and dividing this value into our total load of 7125 lbs. we get a required area of .593 sq. in. This area is approximately equal to a rod having a diameter of 7/8". We will use, however, as tie rods for this tower a diameter of 1", but we will not upset the ends of these tie rods for reason previously stated. When the ends of rod are properly threaded for the clevis and turnbuckles we will have a diameter at the root of the thread

of about 7/8". It might be mentioned here that to prevent undue lateral vibration in these tie rods we will clamp each pair together at their intersection as shown on the accompanying drawing.

ANCHOR DESIGN.

We will now investigate the anchoring of the tower. The total horizontal wind pressure tending to shear the anchor bolts, which will be used to anchor the four columns to the concrete foundations shown on the drawing, is equal to about 20,000 lbs. In this deduction it will be noticed that we have ignored the friction between the bottom of the columns and the concrete foundations. We will use two anchor bolts at each one of the four posts. Hence, we find, the shear per bolt is $= \frac{20000}{8} = 2500$ lbs., taking an allowable shear per square inch $=$ 8000 lbs., and dividing this into the shear of bolt found above we get the number of square inches required per bolt $= \frac{2500}{8000} = .3125$ sq. in. which is about

equivalent to a bolt whose diameter is 5/8".

Now to investigate the anchor bolts for tension we will, of course, consider the case when our tank is empty, as this is the case of maximum tension on the bolts due to the overturning moment of the wind. We will also consider the wind coming in the direction of a diagonal through two posts and centre of figure. For the sake of our design we will also ignore the two posts not in line with this diagonal; then we have, taking moments about fartherest post from the wind , a total wind pressure on the projected area of our tank and tower, times the vertical distance from the centre of this wind pressure to the ground = the weight of the tower times the distance Y (shown on the accompanying diagram) plus the tension on the two bolts in question times the distance 1. Substituting values in the above we find,

$$20{,}000 \times 60 = 50{,}000 \times 14 + T \times 28$$

Therefore T = 17860 lbs.

And we find the tension on each bolt = 8930 lbs.

Taking an allowable tension per sq. in. = 12000 lbs. and dividing this into 8930 we get an area required to hold down the tank of .745 sq. in. which is about equivalent to a bolt whose diameter 1". We will use however, in this design two anchor bolts at each post, each bolt being 5'6" long by 1-1/2" in diameter. The bolts will be provided with a cast iron anchor plate 9" square by 1" thick for anchoring them inthe concrete foundation .

Having determined the size of anchor bolts required to securely ~~rivet~~ anchor our tower to the foundations, we will next consider the design of the bearing plate on the end of each column. We find upon consulting different

authorities that the safe allowable load upon a monolithic cap stone made of granite or of concrete is about 20 tons per sq. ft., hence, dividing our total load on the column of 139000 lbs. by 40000 lbs. we get a required area of 3.5 sq.ft. or 504 sq. in. We will therefore use a bearing plate 22" square. The thickness of the plate is generally taken in columns of this size to be 1" thick. We will therefore use for our bearing plate a plate 1" thick. The detail of securing this bearing plate to the end of column is fully explained on the accompanying drawing.

PIER DESIGN.

The design of the pier or foundation for each one of the columns is determined as follows:-

Have good moderate dry clay; we find that a load of three tons per square foot is allowable. Our column load as above determined is about seventy tons, hence, we find that it

will be necessary to have at least 23. square feet of area to the bottom of each one of the four foundation piers. We will use, however, a pier whose base is 6 feet sq. at the bottom as shown on the accompanying drawing.

We found recently by taking a limiting case that the greatest tension on any two anchor bolts was 17,860 lbs. Now in order to be sure that our tower will not overturn we must provide a weight of concrete greater than the above load. It will be found upon investigating the pier, as detailed in the accompanying drawing, that it has a weight considerably in excess of the required amount. We have also not considered the amount of weight of the earth above this pier which has a considerable holding down value.

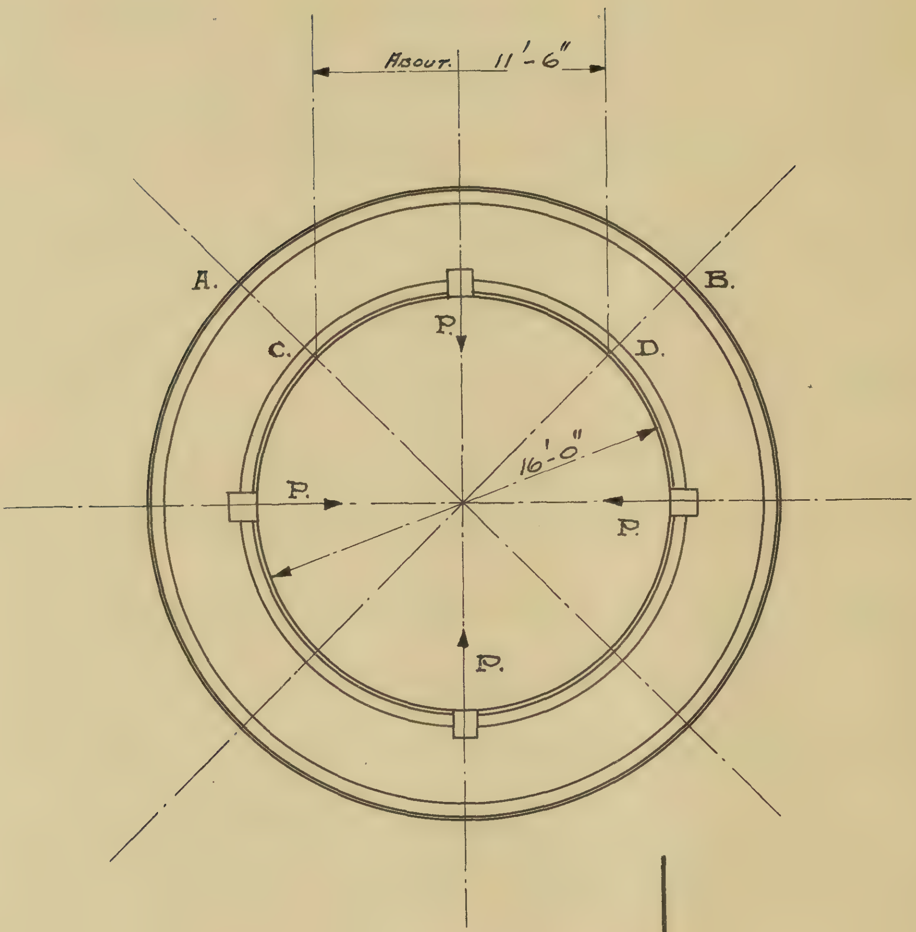
There are yet two points in the design of this structure which we have to consider.

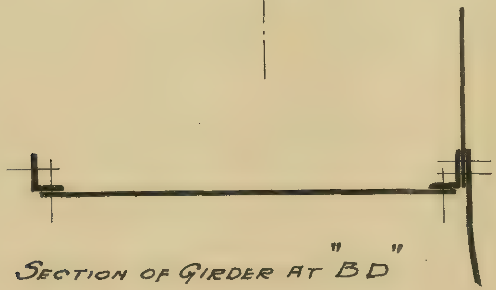
We have designed all of the members of the tower to withstand the wind pressure and the

load of the tank. We have designed the cylindrical portion of the tank together with the hemispherical bottom to withstand the loads imposed, but we have not considered in detail the stresses resulting from the joining of the tank with the tower.

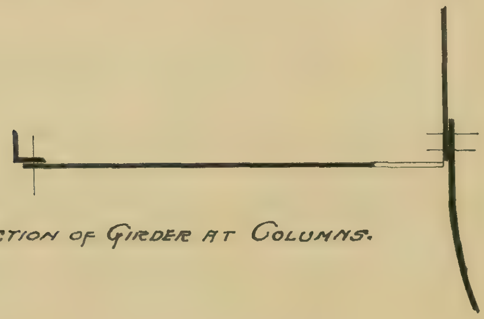
There is a horizontal thrust at the top of each column, as previously explained. Unless there is some means for distributing this thrust uniformly around the circumference of the tank, the tank will collapse. We shall use for this purpose a horizontal plate girder, as will be seen by referring to the accompanying drawing.

The accurate design of this girder is very complicated, and a complete solution of the problem, as applied to the arch bridge, will be found in a book published by Professor Church " Mechanics of Engineering" page 444-449. An adapted design from the solution made by Professor Church will be found in Johnson's





Section of Girder at "BD"



Section of Girder at Columns.

" Modern Frame Structures" page 206-218. There is no reason in the writer's opinion why this design could not be applied to the case in hand. We will, however, in our case in designing this girder, consider a limiting case and from this case determine the size of the web of the girder and the angle irons stiffening the web.

Referring to the accompanying design it will be noticed that we have a concentrated load P acting on the inside of a circular plate girder AB. There are four such loads around the circumference of our tank. We will consider, however, for this design that we have a concentrated load P acting on the centre of a straight beam of length 11'6" and supported at the joints C and D. We believe this assumption is rational and well within the bounds of safety.

P (maximum) = approximately 20,000 lbs. Now using the customary bending moment formula for the design of a beam we have $M = \frac{SI}{c}$

M = 1/4 P x l = 1/4 x 20000 x 11-1/2 x 12 = 69000 lbs. inches.

We will right here assume a section of girder as shown on the accompanying sketch.

Now in order to determine a value of I, that is the combined moment of inertia, we must refer the moment of inertia of each individual member of our section to the neutral axis of the whole. We do this by the use of the old familar formula $I_1 = I_o + Ah^2$.

The value of I_1 for the 4" x 3" x 3/8" angle with ~~the~~ respect~~ive~~ to neutral axis of our girder is found to be 94.21.

Referring now to the other side of our plate girder we find, by examining the method used in attaching the column to the tank, that the top of the column cuts out a square portion of our web 10-1/2" x 10-1/2", and also

cuts off the 4" x 4" x 1/2" angle. But we have to consider as part of our girder, two thicknesses of plate one 3/8" thick and the other one 1/4" thick, which are really plates in the shell of tank. For the sake of this design we will consider these two plates as one cover plate 1/2" thick and 8" wide. Obtaining now a value of its moment of inertia with respect to the neutral axis of the plate girder we find a value of $I_1 = 400.083$.

The moment of inertia of the web itself 1/4" thick x 21" wide is 166.6. The combined moment of inertia with respect to the neutral axis of our plate girder is therefore 660.

We will take c = 10" therefore S, the allowable stress, per square inch = $\frac{69,000 \times 10}{660} =$ 1045.

It would appear from the above discussion that the section of plate girder used on the accompanying drawing is in excess of that

required. But there is another point which we must take into consideration, namely, the loads to which this girder is subjected from a torsional moment which is brought upon it as will be hereinafter explained.

The remaining stress above referred to is a stress in the lower cylindrical plate of the bottom of the tank to which the columns are attached, due to the total weight of the tank and the water in the tank. This load we will consider as coming on this plate uniformly around the circumference of the tank.

In an article written by G. P. Starkweather published in the"Engineering News" November 15, 1900, is a complete discussion of the stresses in a cylinder plate girder. It is a case exactly like the lower cylindrical ring of our tank, which design we will now consider. It will be unnecessary here to quote this discussion in full, but we will use the results

obtained, in our design. Mr. Starkweather proved that in a water tower supported on four posts the stress in the cylinder girder to which the posts are attached gave:-

A maximum negative bending moment over each point of support = .03415 Wr.

A maximum shear in the girder $= \frac{W}{8}$

A positive bending moment midway between the supports = .01762 Wr.

A maximum torsional moment = .0053 Wr.

An angular distance from point of support to point of maximum torsion = $19^\circ\ 12'$

In the above formulas W is the total weight supported by the girder in lbs., and in

our case is equal 361000 lbs., r is the radius of the girder in inches.

The bottom ring of our tank is 5 feet high, and we will assume a thickness of 3/8". Then substituting in the maximum bending moment formula above given we get a value of $M = 1,176,860''$ lbs. And equating this to $\frac{SI}{C}$ and solving for S we get:-

$$S = 5230 \text{ lbs.}$$

This value of S it will be noted is well within the bounds of the allowable unit stress for the grade of steel used in this tank.

Now because of some uncertainty in the determining of the remaining stresses in this girder we shall use the assumed thickness, namely, 3/8", although it would appear that we could use a much lighter plate.

The maximum torsional moment referred

to in a previous paragraph is the torsional moment resulting from the load on this cylinder girder which in our problem is transmitted to the horizontal plate girder whose design we have previously considered, and in which design we have allowed for this torsional moment.

STEEL.

The steel used in this hemispherical bottom tank and its supporting tower will be made by the open hearth process. In explaining the quality of the steel to be used we will start at the top of the tower with the roof.

We will use for this roof a quality of steel known in the trade as "Tank Steel". This is a steel which is made by the open hearth process, but which has no physical or chemical properties guaranteed by the mills who make it. It is apt to be hard and brittle and should never be used where there are specific stresses to be taken care of.

For the cylindrical shell of the tank we will use a grade of steel known as "Medium Steel". This steel is a good grade of steel, but the mills in rolling it make only a heat test. In this test however, they require a

tensile strength of from 60,000 to 70,000 lbs. per square inch.

For the bottom of our tank we will use a grade of steel known in the trade as " Flange Steel ". This steel will have an ultimate stress of from 52,000 to 62,000 lbs. per square inch, an elastic limit not less than one-half the ultimate strength, and an elongation of not less than 25% measured in a length of 8". It will also withstand cold and quench bends 180° flat on itself without fracture on outside of bent portion. For the chemical properties of this grade of steel, we will allow a maximum phosphorus content of .06% and a maximum sulphur content of .04%. It is necessary to make the bottom of flange steel, which is a very soft grade of steel, because of the fact that the bottom will have to be dished, as before mentioned, and if a harder grade of steel were used the process of dishing might injure the plates.

Steel used in the tower supporting the tank and in the angle irons supporting the roof will be made by the open hearth process, as above mentioned, and will have a tensile strength of from 60,000 to 70,000 lbs. per square inch, an elastic limit of not less than one-half the ultimate strength and an elongation of not less than 22%. It will conform to a bending test of 180° to a diameter equal to the thickness of test piece tested without fracture on the outside of bent portion. The maximum phosphorus allowable will be .14%. It is not customary in a steel structure to specify the percentage of sulphur.

In regard to the quality of the rivets used in this structure, we will use for the cylindrical portion of the tank, and also the hemispherical bottom, rivet steel having an ultimate strength of 45,000 to 55,000 lbs. per square inch, an elastic limit not less than one-

half the ultimate strength, and an elongation of 28%, and cold and quenched bends 180° flat on itself without fracture on the outside of bent portion.

For the rivets in the tower and the structural work in general, it will be necessary to use a soft grade of steel, but we will use a rivet steel having an ultimate strength of 48,000 to 58,000 lbs. per square inch, an elastic limit not less than one-half the ultimate strength and an elongation of not less than 26% and a bending test of 180° flat on itself without fracture on the outside of bent portion.

It has been found necessary in structures of this kind to use the above different qualities of steel for each specific purpose because of the fact that there is such a wide difference in the price of the different grades of steel.

PAINTING.

The question of protecting steel work before and after erection is too often neglected in engineering work. It is neglected perhaps by the Engineer because he thinks that it is a subject not in his sphere. The writer has in his search for light on the subject of painting found the most valuable data in a Committee Report submitted by the " American Society for Testing Materials". The report is too long to include in this thesis however. The conclusions in J. N. Hazelhurst's chapter on painting are brief and to the point and are here presented as they embody the good features in above mentioned report.-

" To sum up, in favor of the use of red lead and oil is its well-known high specific gravity and its peculiar chemical property of combination, resulting in the production of a coating or film of a particularly tenacious, hard, and insoluble character, when not subject to great

heat or sulphurous gases, which is seldom to be considered in connection with such structures as towers and tanks. The red-lead paint, however, lacks elasticity, resulting in the formation of air-cracks, and its porosity from the escape of volatile gases during the process of hardening seems to be well established. Moreover, its high specific gravity has the disadvantage of causing the pigsment to"sag" or run away from the oil when being applied, resulting in streaking or imperfect and uneven covering, while its quick-setting qualities render this paint unsatisfactory and difficult to handle. The last tendency may be in part or entirely removed by the addition of the mixture of carbon, usually in the form of lampblack, which further aids, as has been shown, in diminishing the porosity offered as an objection to the use of lead and oil, while if paint is used, as before erection, upon materials and surfaces which may be placed horizontally or

nearly so, the pigment has little or no opportunity to settle out of the oil or "sag".

In the manufacture of such paint it is a prime necessity that, to produce satisfactory results, each ingredient should be chemically pure, and the degree of purity will determine the relative efficiency. Suitable proportions have been found in 20 lbs. of red lead, 1 lb. of carbon as lampblack to 5 or 6 lbs. of raw linseed oil. The bulk will be about 1 gallon, with a covering capacity of about 50 square yards of surface for the first coat, the film being approximately .002 of an inch in thickness. The cost will be about $1.50, and the amount paid for labor in spreading will run about 5 cents per square yard where the services of an experienced painter are employed.

For all the reasons submitted, it would appear that as a priming coat, or first coat, red lead, lampblack, and linseed-oil, when applied upon

iron or steel surfaces of structural material before erection, affords the best known protection to metallic corrosion; it is also a well established fact that red lead, usually as a red-lead paste, is used in water and steam-pipe fitting to produce a close and perfect joint, and that when applied upon the laps of steel plate intended to be used in water-tank construction, the same tendency toward producing a water-tight joint is observed, and the use of this material for such purposes minimizes the most objectionable practice of making it necessary to resort to a natural or rust-joint to secure the necessary degree of tightness between the metal plates.

It also seems equally sure that suitable finishing coats should be provided and applied over the priming coat, and that this last film should be of small specific gravity, elastic, impervious to moisture, hard, and tenacious, it should be indifferent to sulphurous gases and

electrically insulating, all of which properties seem to be fulfilled to a greater degree by an asphaltic varnish than any known varnish or paint composition. On account of its ease of application and quick-drying powers, it is particularly suitable for application upon structures being erected in the open air and exposed to the weather, while the characteristic of a volatile composition to produce a deposition of moisture is of no consequence when that moisture is not formed upon the metal itself, but upon a cement-like coating, which, besides, has a power for decomposing moisture by the absorption of its oxygen.

The cost of a well prepared asphaltic varnish, of pure materials, will be about $1.50 per gallon, which will cover about 40 square yards of surface, one coat.

Either paint or varnish coats should, when possible, be put on under the most favorable atmospheric conditions, the best season being

during the autumn, when the temperature is apt to remain more uniform, and when fogs and rains are less likely to occur. A suitable interval of time should be observed in order that the first coat should be entirely and completely dry before the second coat is added. In order to have the painter or contractor observe this, and to make sure that more than one coat is put on, the several coats should differ slightly in color, so that such neglect would be readily determined and corrected.

In the purchase of materials, the preference should be given old and long-established houses, whose reputation for quality is well-known, and it should not be expected that the purchase of paint materials at less than market prices will be conducive of anything but the practice of adulterating the products.

In the application of the paints, which should have been selected with considerable care

only experienced and reliable mechanics should be employed; in the long run, besides their ability to spread a smooth and regular coat, their experience will save sufficient material, or make the same material go enough further, to warrant the employment of skilled mechanic, if the selection of the individual is put upon a basis of first cost, rather than of comparative excellence.

Interesting data concerning the amount of paint required per unit of surface is here tabulated:-

Average Surface Covered Per Gallon Of Paint

Character	Volume of Oil.	Lbs. of Pigment.	Volume and Weight of Paint.		Sq. Ft. 1 Coat.	2 Coats.	Cost per Gal. Mixed.
			Gals.	Lbs.			
Red lead (powdered)	1. gal.	22.40	1.4	30.40	630.	375.	1.50
Graphite (in oil)	1. "	12.50	2.0	20.50	360.	215.	1.00
Black asphalt	1. "	17.25	4.0	30.00	515.	310.	.90
Asphaltic varnish	1. "		1.0	8.00	400.	300.	1.50
Pure linseed-oil	1. "				875.		0.50

CONCLUSION.

We have now completed the essential features in the design of this hemispherical bottom tank and its supporting structure. There are some details which we have not considered, however, because of the fact that they are structural details and they can be studied on the drawing which the writer has endeavored to make very clear.

It was the intention, when this thesis was started, to discuss at some length the shop methods used in the building of this tank, and also the methods used in erecting it in the field, but as it has reached rather voluminous proportions, this discussion will be omitted.

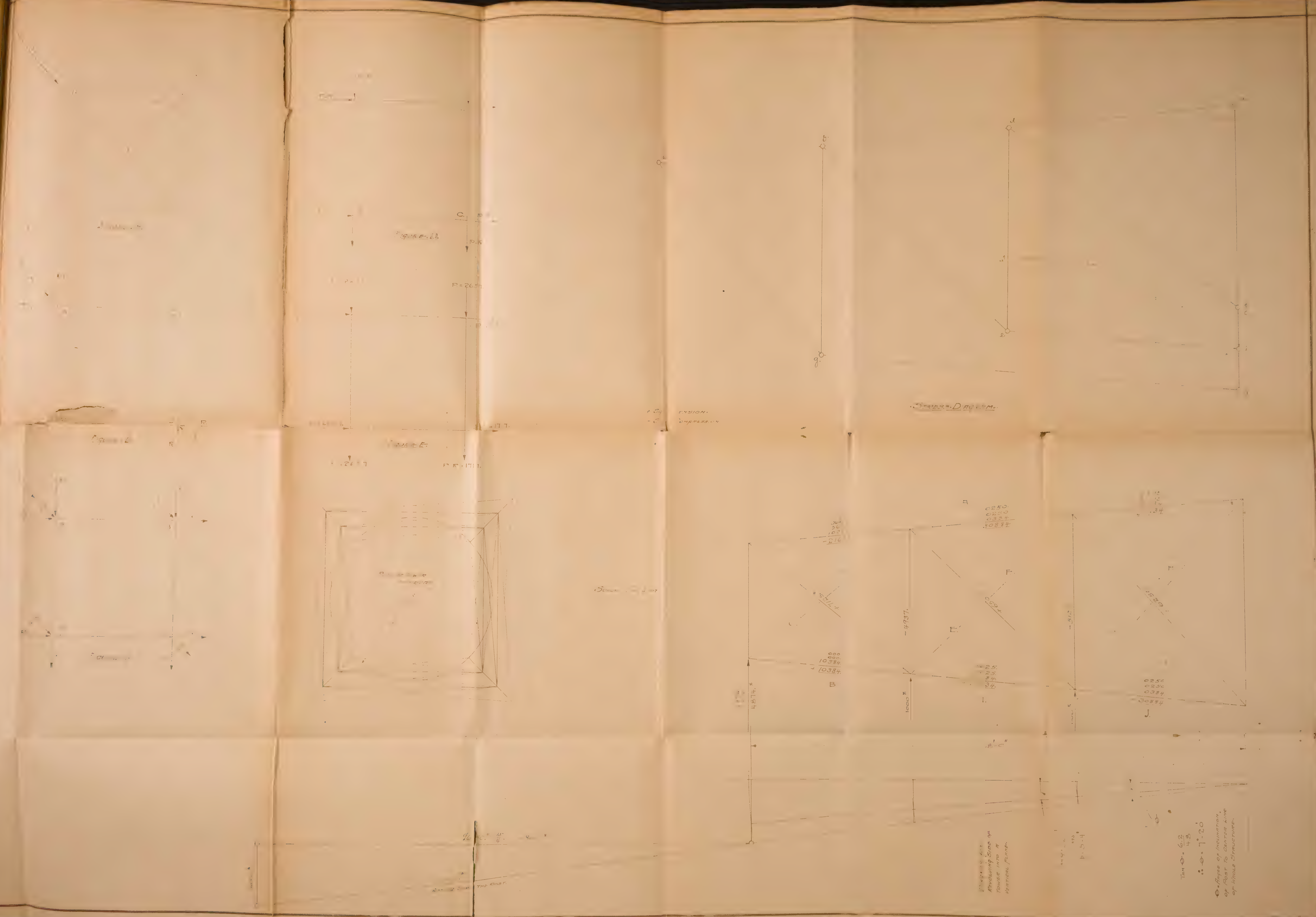
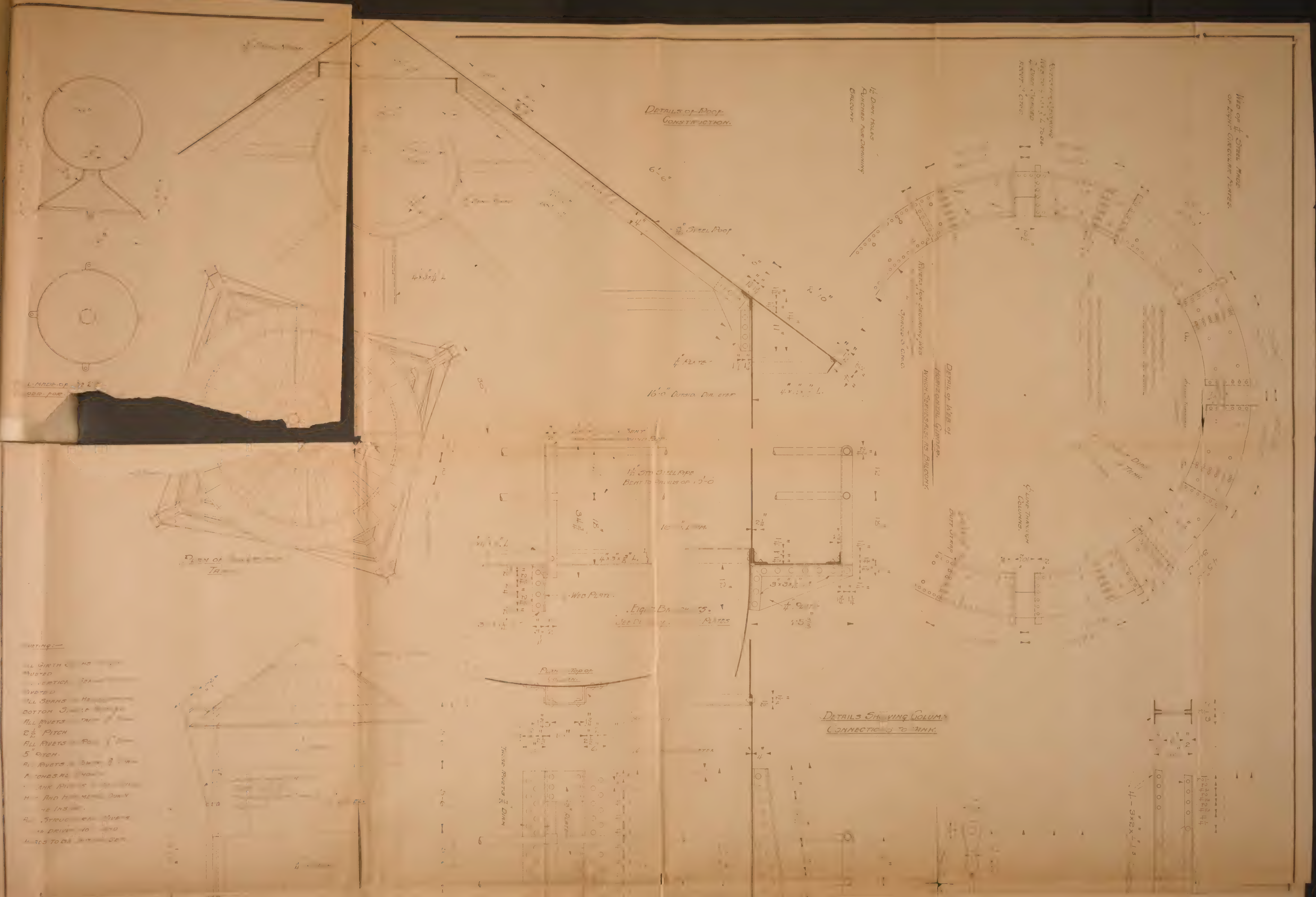
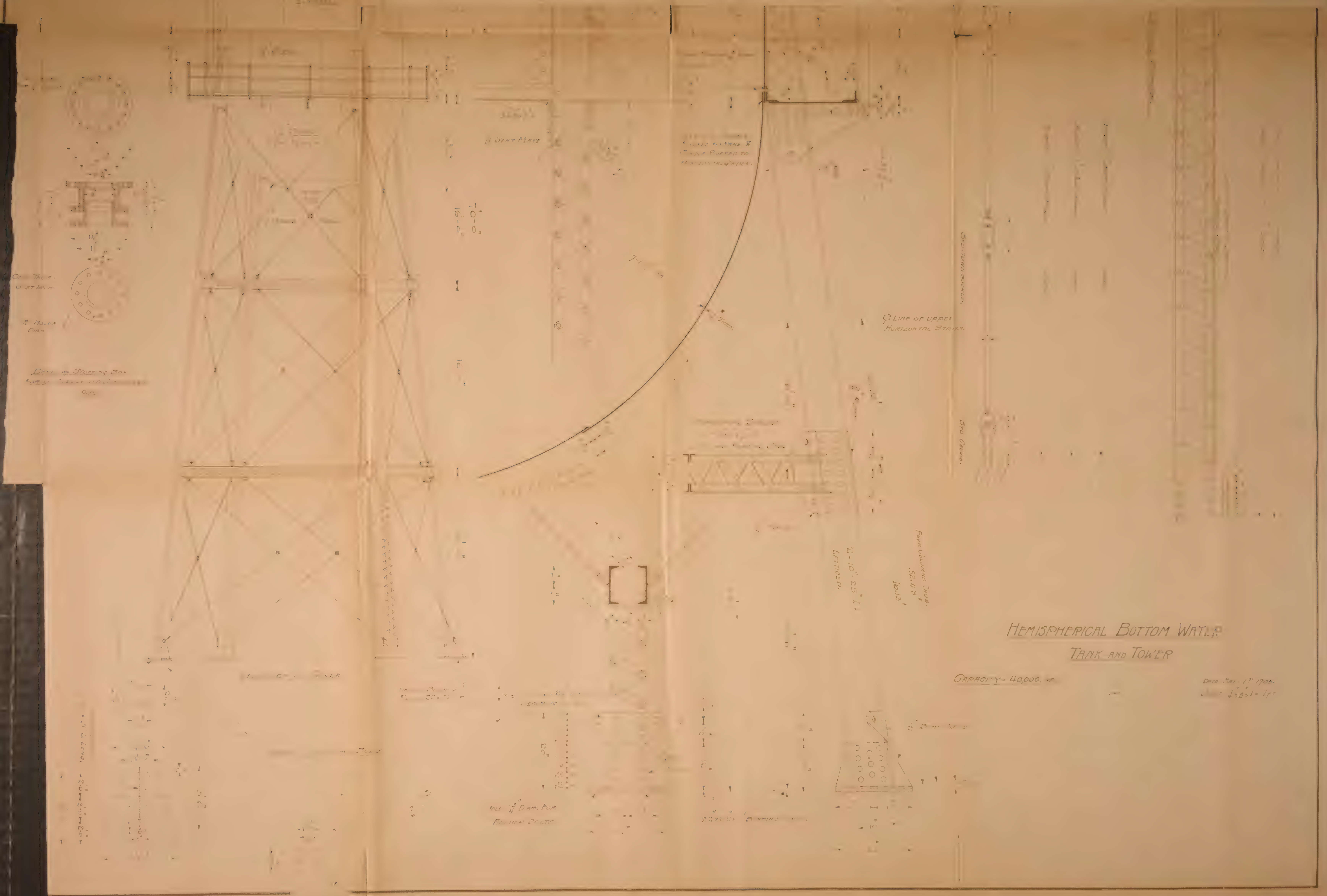


Figure D.
C.
P.K.
P = 26.57.
= 17 7.
Figure E.
= 26 7
P.K. = 17 11.
+ Sig. Tension.
- S. Compression.
g.
J.
2.
Stress Diagram.
-216
+7135.
000
000.
10384.
+10384.
B
4874.
A
0250.
0250
0384
30884
F.
+659
-4937.
E.
1000
25.
25.
84.
34.
I.
J.
0250
0250
0384
30884
8'-0"
Revolving Side of Tower into a Vertical Plane.
p = 5-4
Tan Φ = 6.2/4.8
∴ Φ = 7°-20'
Φ = Angle of Inclination of Post to Center Line of Whole Structure.

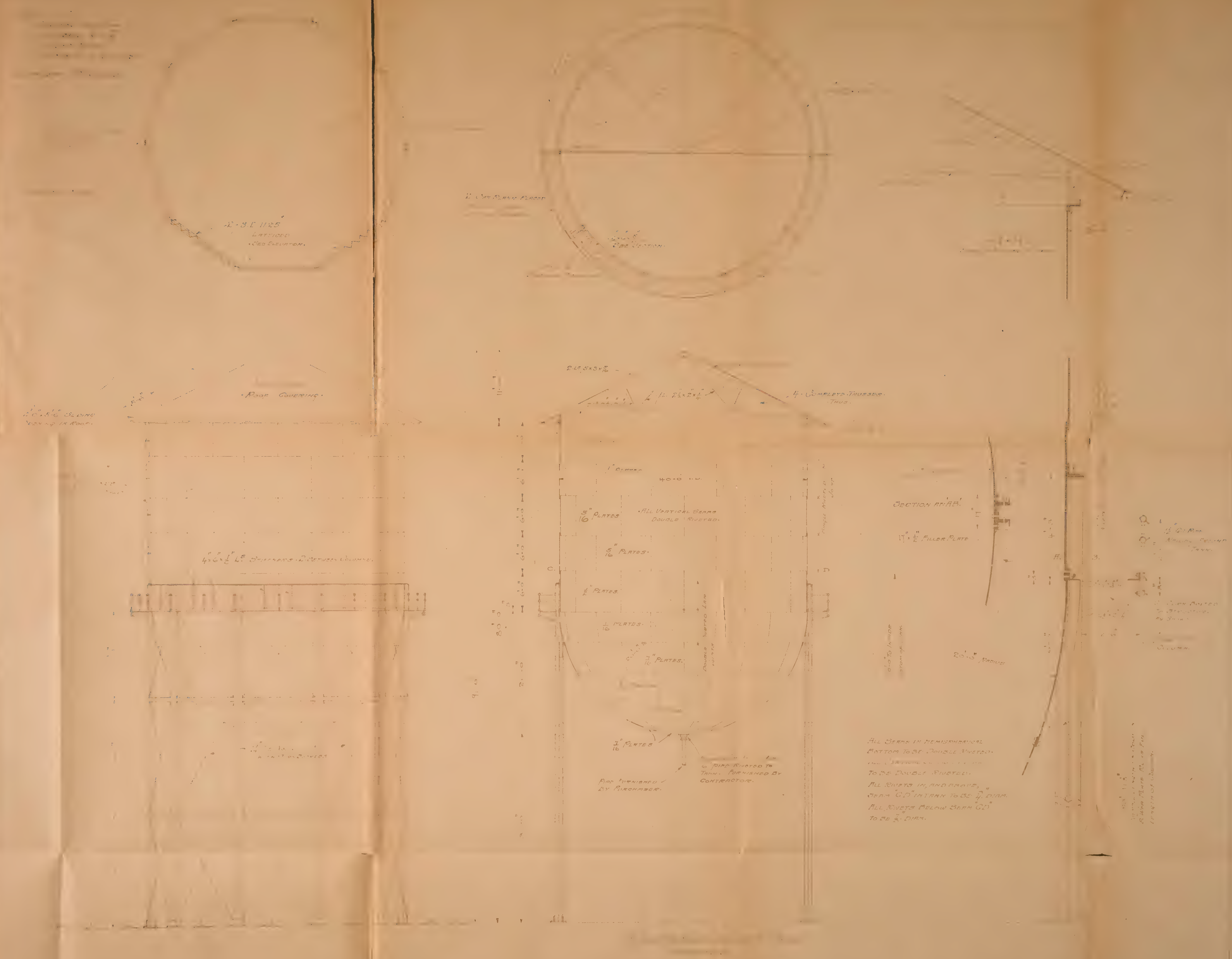

DETAILS OF ROOF CONSTRUCTION.
5/16" STEEL ROOF
4"x3"x5/16" L
1/4" PLATE
16'-0" OUTSIDE DIA. STEP
4"x3"x3/8" L
6'-6"
1 1/2" STD. STEEL PIPE
BENT TO RADIUS OF 13'-0"
3 4 1/2"
18"
4"x3"x3/8" L
WEB PLATE
EIGHT BRACKETS.
3"x3"x1/2" L
1/4" PLATE
PLAN OF TOP OF COLUMN.
TWO RIVETS 3/4" DIAM.
PLAN OF ... TANK
DETAILS SHOWING COLUMN CONNECTIONS TO TANK.
4-3"x2"x1/2" L
1/2" DIAM. HOLES
PUNCHED FOR DRAINING
BALCONY.
WEB OF 1/4" STEEL MADE
OF EIGHT CIRCULAR PLATES.
DETAIL OF WEB OF
HORIZONTAL GIRDER
WHICH SERVES ALSO AS BALCONY.
RIVETS FOR SECURING WEB
SPACED 3" CTRS.
DUTT STRAP
¢ LINE THROUGH COLUMNS.
DIA. TANK.
45'-0"
30'-0"
10 1/2"
L MADE OF
RIVETING:—
ALL GIRTH SEAMS
RIVETED
VERTICAL SEAMS
RIVETED
ALL SEAMS HE
BOTTOM SINGLE
ALL RIVETS
2 1/2" PITCH
ALL RIVETS PO
5" PITCH.
ALL RIVETS 3/4"
PITCHES AS SHOWN
TANK RIVETS
AND HE ME
HE INSIDE.
ALL STRUCTURAL RIVETS
TO BE DRIVEN HOT AND
HEADS TO BE



HEMISPHERICAL BOTTOM WATER
TANK AND TOWER

CAPACITY = 40,000

Date May 1st 1900.



ALL SEAMS IN HEMISPHERICAL
BOTTOM TO BE · DOUBLE RIVETED ·
[illegible]
TO BE DOUBLE RIVETED.
ALL RIVETS IN, AND ABOVE,
SEAM "CD" IN TANK TO BE ¾" DIAM.
ALL RIVETS BELOW SEAM "CD"
TO BE ⅝" · DIAM.

· COATESVILLE · BO[illegible]

SCALE ⅛" & ½" = 1 FT.

DR'Y.

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